Chapter 3
Embankment Construction Problems

The analysis of embankment construction involves the estimation of stresses and movements in embankments and their foundations both during and after construction. The construction of embankments generally involves both excavation and filling in some specified order. The finite element method offers an ideal way to perform such an analysis because of its ability to handle complex geometries, construction sequences, and nonlinear soil behavior. Some examples of embankment construction problems include those in embankment dams, levees, and highway embankments. Additionally, a static finite element analysis is often performed as part of the evaluation of liquefaction potential of foundation soils beneath an embankment as the cyclic strength of soil depends on the state of stress in the soil (Wahl et al. 1987).

3-1. Results and Use of Embankment Construction Analysis

a. *Embankment and foundation system*. The stresses and movements obtained from the analysis can be used to evaluate the expected performance of the embankment-foundation system against pre determined performance standards. The finite element analysis should be used in conjunction with a conventional slope stability stability program to ensure that both give the same results with respect to the stability of the system.

b. *Finite element analysis*. The finite element analysis can be a useful tool during the design process of an embankment. Parametric studies can be performed for the purpose of dealing with uncertainty in the foundation conditions and material properties. The results of these studies can provide a range of values for stresses and movements which can be compared with allowable values to help ensure the adequacy of the design.

c. *Construction process*. The finite element analysis of an embankment can also be useful in the construction process, since it can serve as an aid in the selection of the types and locations of in-stumentation systems that monitor performance both during and after construction. This type of analysis can also provide insight into the interpretation of movements and distribution of stress in the embankment-foundation system based on data collected from settlement gages, slope indicators, pore pressure transducers, etc.

3-2. Important Features of Embankment Construction Analysis

The following paragraphs describe several items and considerations necessary for the performace of a good finite element model for embankment construction:

a. *Material behavior models*. Soil is the primary material of construction in embankment construction problems. As described in the introduction of this ETL, the stress-strain behavior of soil is nonlinear and inelastic. For all cases except saturated soil under undrained conditions, the stress-strain behavior of soil is dependent on confining pressure. These aspects of soil behavior are encountered in most geotechnical engineering projects, including projects involving the construction of embankments. Consequently, it is important that the material model be capable of tracking these aspects of soil behavior. Many material models, such as the hyperbolic model of Duncan and Chang (1970) and the Cam-Clay model (Roscoe and Burland 1968), do capture these characteristics of soils. The hyperbolic model uses a confining pressure-dependent, nonlinear elastic formulation, with an inelastic component introduced, because the value of the unload-reload modulus is larger than the value of the virgin loading modulus. The Cam-Clay model uses a plasticity formulation that also yields reduced modulus values as the soil strength becomes mobilized and increased modulus values as the confining pressure increases. One of the key benefits of plasticity is that it can model plastic strains that occur in directions other than the direction of the applied stress increment. This feature becomes especially important when a soil mass is near failure. In such a case, the application of a load increment in one direction can cause large displacements of the soil in another direction if large forces had been previously applied in that other direction. For well-designed structures, in which failure of large masses of soil is not imminent, modeling this aspect of failure can become less important.

b. *Stress-strain material properties values*. Selection of appropriate stress-strain material property values is often the most important step in performing SSI analyses. There are four methods to obtain material property values:

(1) Sampling and laboratory testing. For foundation soils, relatively undisturbed samples should be obtained. For embankment or backfill materials, laboratory compacted specimens can be prepared. In either case, the specimens should be tested in the laboratory in an appropriate manner to obtain the necessary parameter values for the material model that will be used. Typical laboratory tests for obtaining these values are 1-D consolidation tests, isotropic consolidation tests, triaxial compression tests, and direct simple shear tests.

(2) Field testing. Some *in situ* tests, e.g., the borehole pressuremeter tests can be performed to obtain material property values.

(3) Correlations with index property values. Stress-strain material property values for several soils have been published together with index property values for the same soils, e.g., Duncan et al. (1980). These published values, together with judgment and experience, can be used to estimate appropriate stress-strain material property values based on index property test results for the soils of interest.

(4) Calibration studies. In many cases, designers have experience with local soils and are skilled at calculating 1-D consolidation settlements using conventional procedures. It is good practice in such cases to develop a 1-D column of finite elements that models the soil profile at the site of interest. The 1-D column can be loaded and the resulting settlements compared to those calculated using conventional procedures. The material property values for the finite elment analyses can be adjusted until a match is obtained. Similarly, if an independent estimate of the lateral load response, i.e., the Poisson effect, can be made, the material property values can be adjusted until the 1-D column results match the independent estimate. Ideally, one set of material property values would be found that provides a match to both the compressiblity and the lateral load response over the range of applied loads in the problem to be analyzed. The selection of a method to obtain material property values depends, of course, on the type of information available. These methods are most effective when used in combination.

c. *Finite element mesh*. The finite element mesh should reflect the geometry of the embankment, both with respect to the external surface geometry and the distribution of materials in the embankment and underlying foundation. Additionally, the mesh should reflect the configuration of any excavations or filling operations performed as part of construction. Most embankment construction problems are either 2-D plane-strain or 3-D type analyses. Levees or embankment dams constructed across broad alluvial valleys are good candidates for 2-D plane strain analysis, whereas embankment dams constructed within narrow canyons are good candidates for a 3-D finite element analysis. The mesh should also extend beyond the area of interest until a known boundary condition is encountered (e.g., bedrock can often be represented as a fixed boundary condition) or for a sufficient distance that conditions at the boundary do not significantly influence the calculated stresses and deformations in the area of interest.

d. *Construction sequence*. It is important to model the construction sequence in embankment problems for two reasons:

(1) Soil response is nonlinear.

(2) Geometry can change during construction, e.g., fill placement.

Because the nonlinear stress-strain behavior of soils depends on the confining pressure, it is almost always necessary to first calculate the initial *in situ* stresses in the foundation materials. Perhaps the only exception occurs when a rock foundation is being modeled as linear elastic. In addition, it is necessary to model the following types of construction operations in steps: excavation, fill placement, placement and removal of structural components, and application of loads and pressures. The construction steps should be modeled in the actual order in which they are to be carried out.

e. *Calibration of the entire model*. As can be seen from the foregoing, there are several factors that must be carefully considered to develop a good finite element model of an embankment construction problem. It is important to successful application of the method over the years to calibrate the entire process against instrumented case histories. Fortunately, several such comparisons have been published. Several of these are listed among the references in Chapter 4.

3-3. Case History: Birch Dam

a. *Project description*. Birch Dam, built across Birch Creek between 1974 and 1976, has a maximum height of 70 ft and a crest length of 3,200 ft. The embankment was constructed across alluvial soil deposits which vary in thickness from 10-ft near the abutments to a maximum of 37-ft near the center of the valley. A cross-sectional view of the embankment is shown in Figure 16. The foundation was primarily composed of compressible silts and clays with numerous lenses of silty and clayey sands. The core and cutoff trench contain materials which clas-sify as a CL (according to the Unified Soil Classification System). The upstream and downstream shells contain coarser and less plastic materials which clas-sify as ML's. The finite element analysis of Birch Dam was reported by Soriano, Duncan, and Simon in 1976.

b. *Purposes*. The finite element study of Birch Dam was performed to predict the stresses and movements in the embankment and foundation during construction, at the end of construction, and after filling of the reservoir. The finite element analysis of Birch Dam was reported by Soriano et al. (1976).

c. *Material model, properties, and finite element Code*. The hyperbolic model as implemented into ISBILD (predecessor to FEADAM), was used for the analysis of Birch Dam. The parameters for the soil model were obtained from the interpretation of tests performed in the drained and undrained triaxial tests, respectively. All time dependent stresses and movements were computed *indirectly* since ISBILD is a statics program which does not account for consolidation. Separate finite element analyses were performed to model the construction sequence for both *drained* and *undrained* conditions. These are extreme conditions in which the analysis is carried out assuming that there is no dissipation of pore pressure at all times for the undrained case and complete dissipation of porewater pressures for the drained case. In this study, the authors contrived a scheme based on Terzaghi's theory of consolidation to weight the drained and undrained cases to determine the displacements and stresses in the embankment at any time.

d. *Mesh details*. The mesh used for both the drained and undrained analyses is shown in Figure 17. Only half of the mesh upstream of the centerline was modeled in the analysis due to the symmetrical geometry of the cross section. A full mesh was used to model the filling of the reservoir because of the asymmetry of the loading conditions. Seepage forces were determined from a seepage analysis and applied as concentrated forces to the appropriate nodal points in the full mesh. The resulting movements and stresses were then calculated.

e. *Construction sequence*. The construction schedule is presented in Figure 18. Both the drained

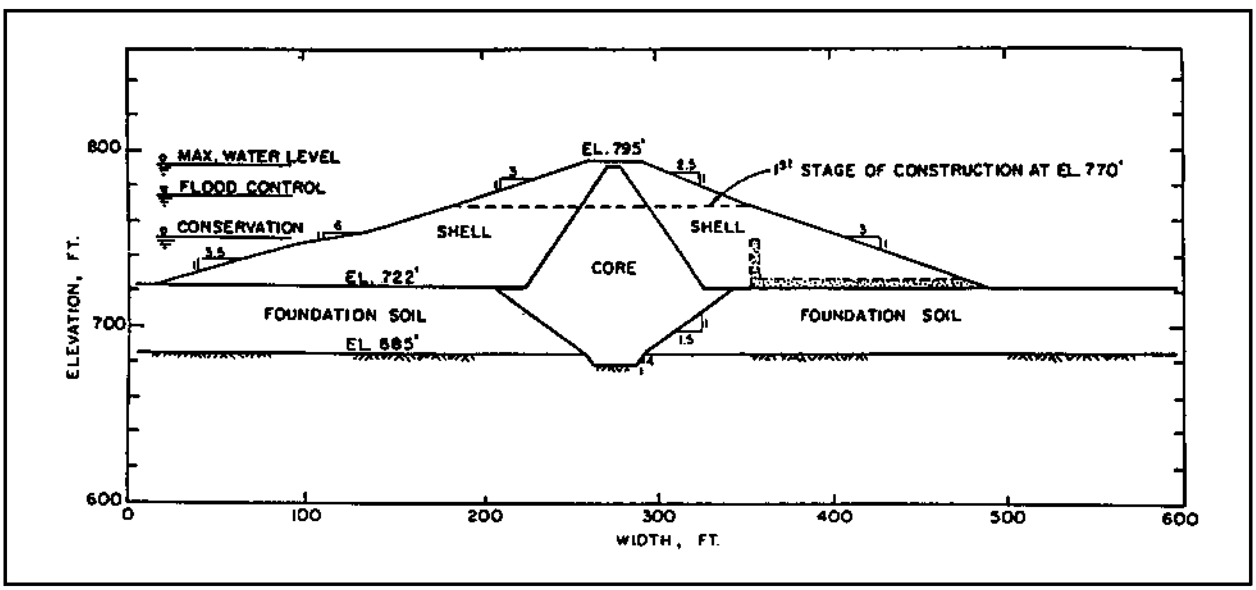

Figure 16. Cross-sectional view of Birch Dam

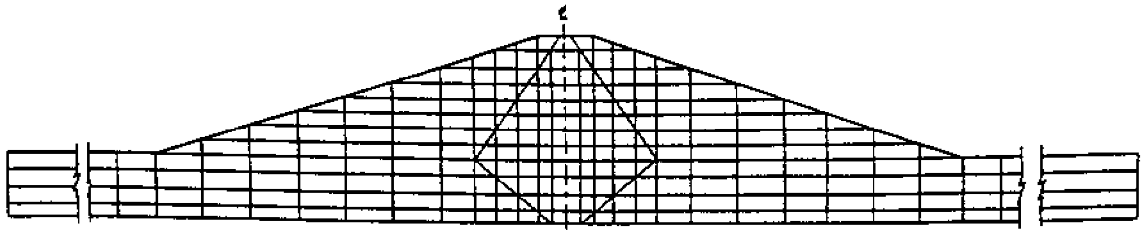

Figure 17. Finite element mesh of Birch Dam

and undrained analyses proceeded according to this schedule. Level ground conditions were presumed to exist just prior to the start of Stage I. The core trench was presumed to be in place at this time as excavation and filling of the core trench were not modeled. As shown in Figure 18, Stage I was modeled in five load steps by placement of the bottom five rows of elements and Stage II was modeled in three load steps by the top three rows of the embankment. Movements and stresses in the embankment and foundation were desired at the following times from the output:

(1) Start of construction, t = 0 months.

(2) End of Stage I, t = 4 months.

(3) End of the waiting period between Stages I and II, t = 13.5 months.

(4) End of Stage II, t = 16.5 months.

(5) After construction had been completed for 13.5 months, t = 30 months.

(6) After reservoir filling.

f. *Results*. A vector plot showing the displacements at various times with consideration of the effects of consolidation is shown in Figure 19. The percentage of the available shear strength mobilized in the cross section for the undrained and drained cases is shown in Figures 20 and 21, respectively. Also shown in these figures is the safety factor of the critical circles from a conventional limit equilibrium analyses. Figure 20 shows that the results for the undrained finite element method analysis agree with those from the slope stability analysis. In these analyses, the critical circle (whose factor of safety equals 1.25) passes through the zone of foundation materials where 100 percent of the available shear strength is mobilized. Similar results are shown in Figure 21 for the drained case where the limit equilibrium analysis showed the safety factor to be 2.33. The estimated horizontal and vertical movements in the embankment and foundation with consolidation taken into account are shown in Figures 22 and 23 at the indicated times. These results are presented in a form consistent with that of data to be collected from instrumentation.

3-4. Case History: New Melones Dam

a. *Project description*. A second example of a finite element analysis of the construction and performance of an embankment dam was reported by Chang and Duncan (1977) for New Melones Dam. New Melones Dam was constructed by the U.S. Army Engineer District, Sacramento, on the Stanislaus River to create a multipurpose reservoir capable of impounding 2.4 million acre-ft of water. The dam, built in a canyon, has a maximum height of 625 ft above the streambed and a length of 1,600 ft. Plan and cross-sectional views of New Melones Dam are shown in Figures 24 and 25.

b. *Purpose*. The purpose of the analysis was to provide insight into three important questions related to the consolidation of the core and behavior of zoned embankment dams. The questions were:

(1) What is the nature of expected movements in a zoned dam during the consolidation of the core?

(2) How do the stresses in the embankment change during consolidation?

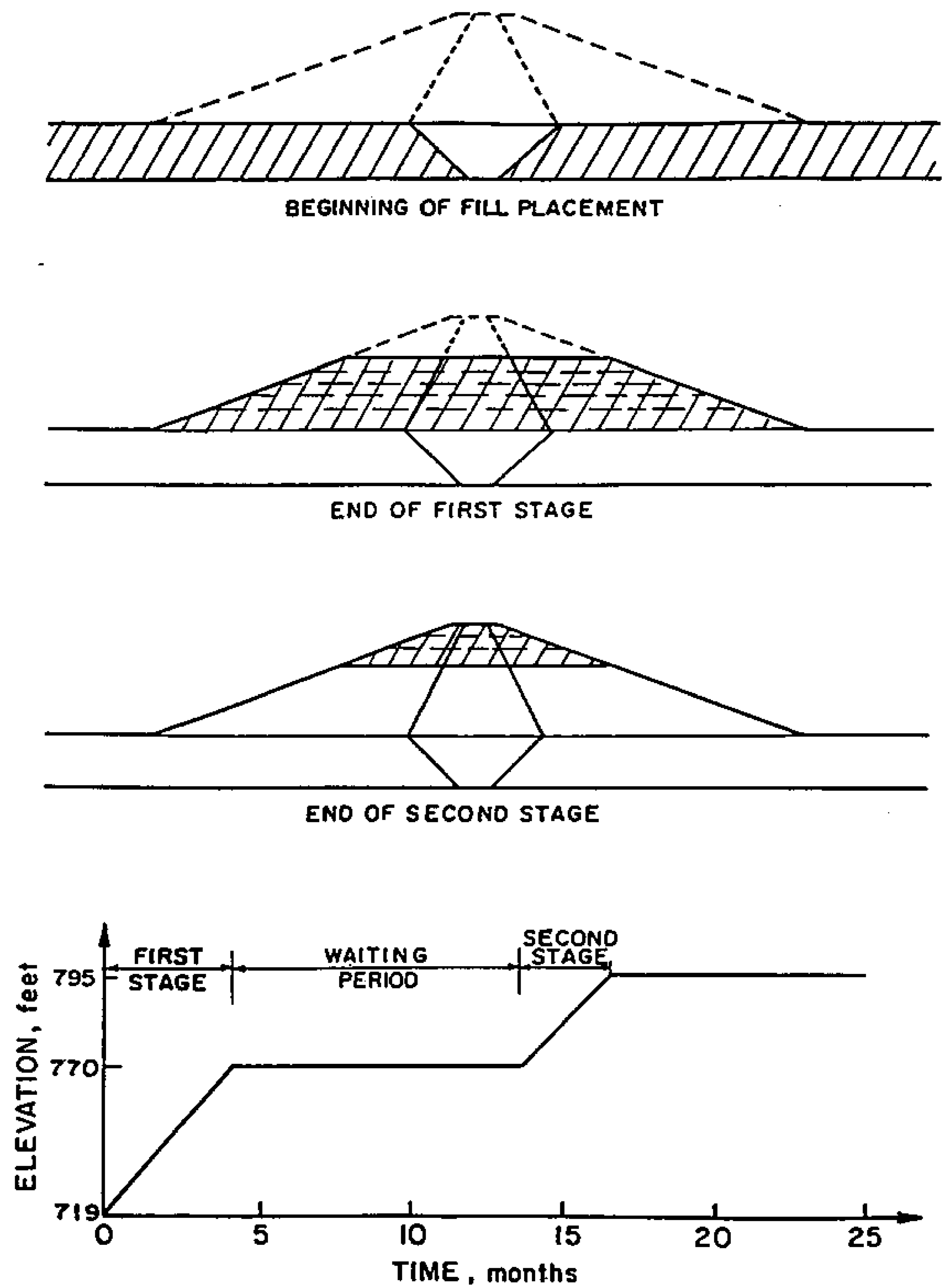

Figure 18. Construction sequence for Birch Dam

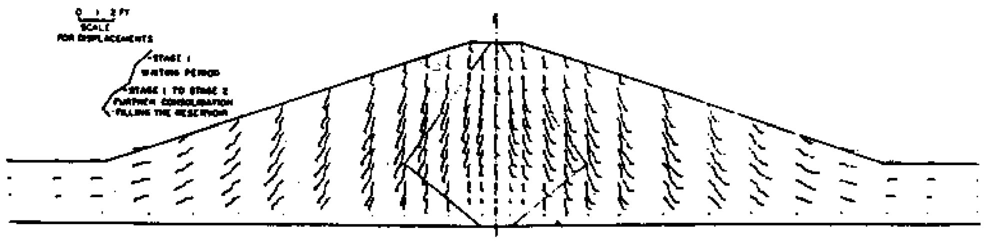

Figure 19. Displacements at selected times

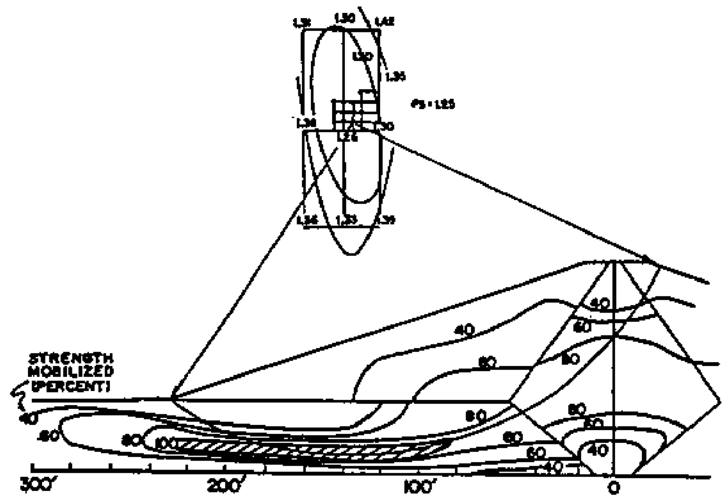

Figure 20. Percentages of mobilized shear strength for undrained case

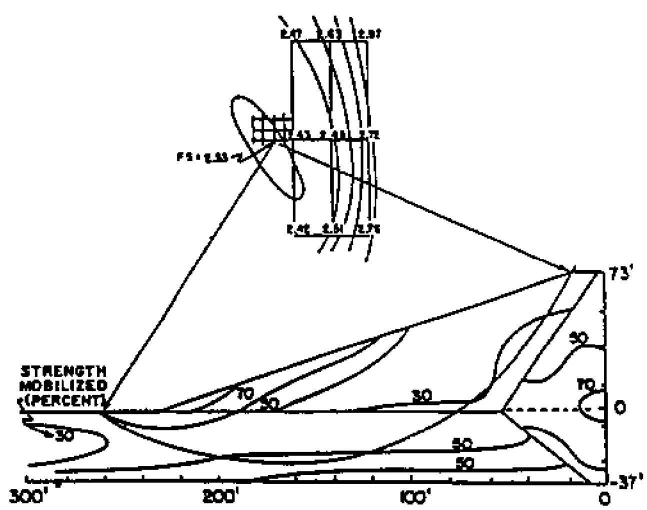

Figure 21. Percentages of mobilized shear strength and critical circle for drained analysis

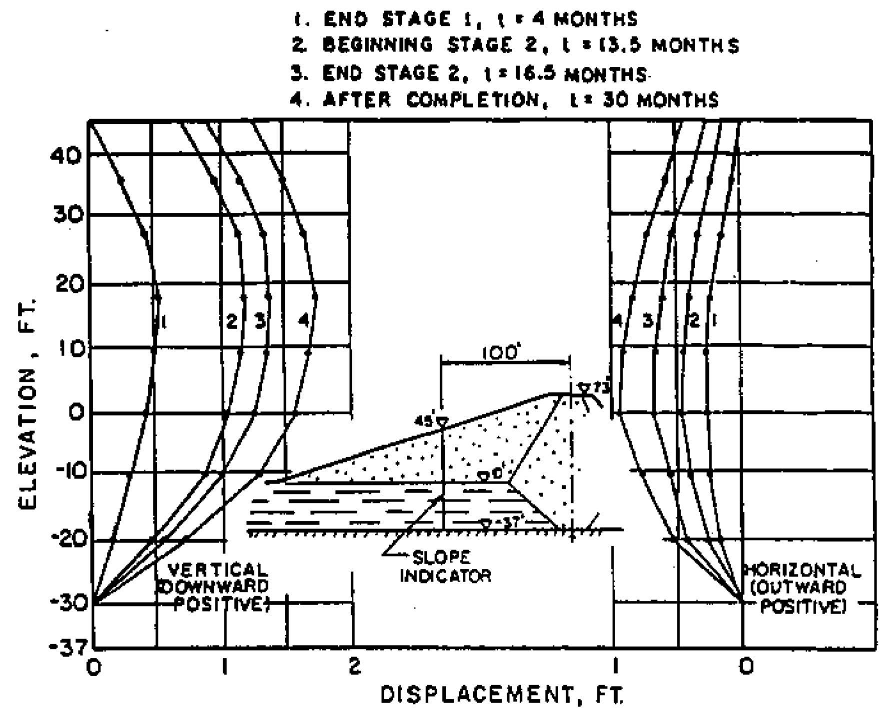

Figure 22. Estimated vertical and horizontal movements of slope indicators at selected times

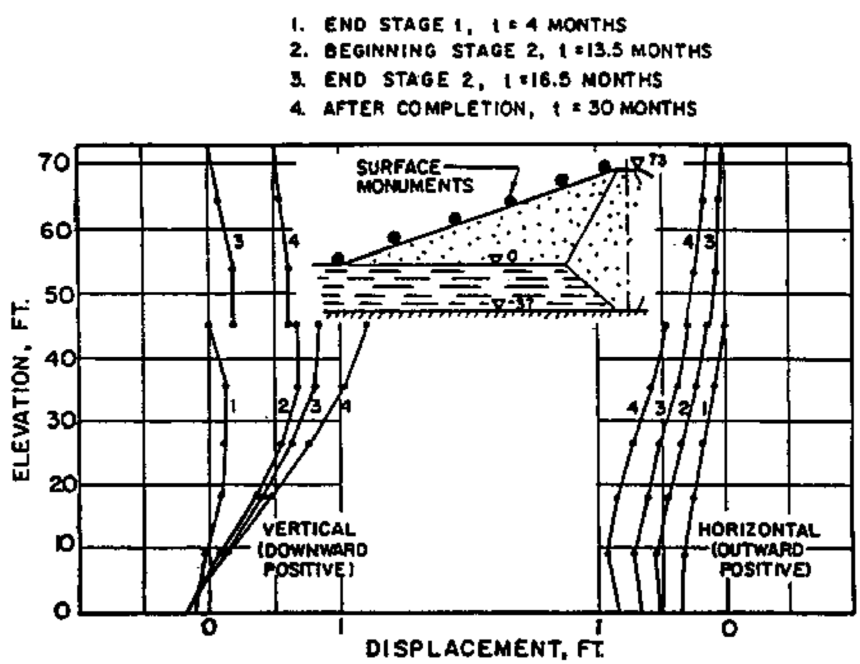

Figure 23. Estimated vertical and horizontal movements of the surface monuments at selected times

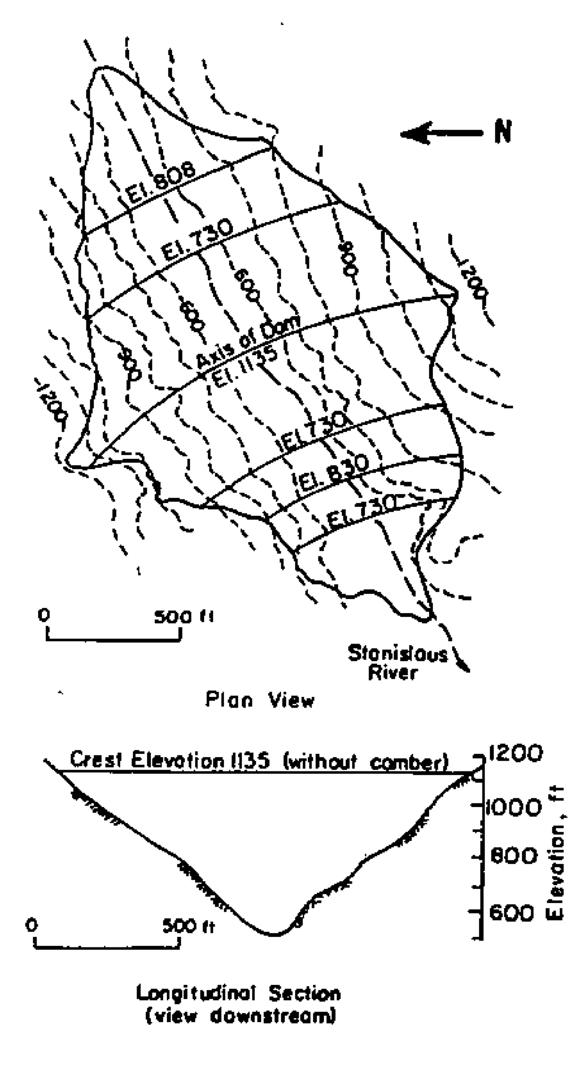

Figure 24. Plan and longitudinal views of New Melones Dam

(3) Are the long-term stresses, calculated assuming slow construction and assuming no excess pore water pressure, the same as those calculated taking into account the effects of consolidation?

c. *Material model, properties, and finite element code*. The finite element code used in this analysis was CON2D, and it has the ability to directly account for the effects of consolidation. In the study, it was assumed that a plane-strain analysis would serve as a reasonable approximation of the performance of the dam in the center of the valley. The mesh is shown in Figure 26. Nonlinear stress-strain behavior was simulated using the Modified Cam-Clay model. Also, in the analysis, the permeability of the shell (rockfill) was very high compared to that of the core ($k = 10^{-7}$ cm/sec). The consolidation of partially saturated soils in the core was simulated using a "homogenized" pore fluid to account for the effects of water and air in the void spaces. Separate analyses were conducted for two different core conditions to account for the variations in water content and dry density which may occur during construction. These analyses accounted for a "stiff" core (corresponding to 95 percent relative compaction as determined by the Standard AASHO compaction test and 1 percent dry of the optimum water content) and a "soft" core (corresponding to 90 percent relative compaction and optimum moisture content at the time of placement).

d. *Construction sequence*. The analysis was performed in three principal stages: (1) construction, (2) reservoir filling, and (3) long-term seepage. The construction represented a timespan of 3.5 years. The construction of the cofferdam was accomplished by introducing elements 1 though 16 as fill in two layers (Figure 26). The remainder of the dam was constructed by the addition of five layers of additional "fill elements." The construction was an undrained analysis as it was assumed that excess pore water pressures did not dissipate during the construction process due to the relatively short timespan of the construction period. The filling of the reservoir was modeled by the application of the water pressures of the full reservoir at the interfaces between the upstream and the core and the impervious soil in the cofferdam zone. The reservoir was assumed to be filled to elevation 990 ft. Forces were applied to nodes connected to "shell" elements to account for buoyancy due to submergence. The application of these pressures and forces is illustrated in Figure 27. Dissipations of excess porewater pressures during reservoir filling were also neglected because it was presumed that the reservoir would be filled within a relatively short period. During the third stage, the long-term seepage stage, fluctuations in the pool level were ignored as it was assumed that the elevation of the pool remained at 990 ft. During this stage, as steady seepage was approached, deformations within the dam were influenced by the dissipation of excess porewater pressures and seepage through the dam. In the analysis, stresses, strains, and porewater pressures were calculated 5, 15, 50, and 80 years after the reservoir was filled.

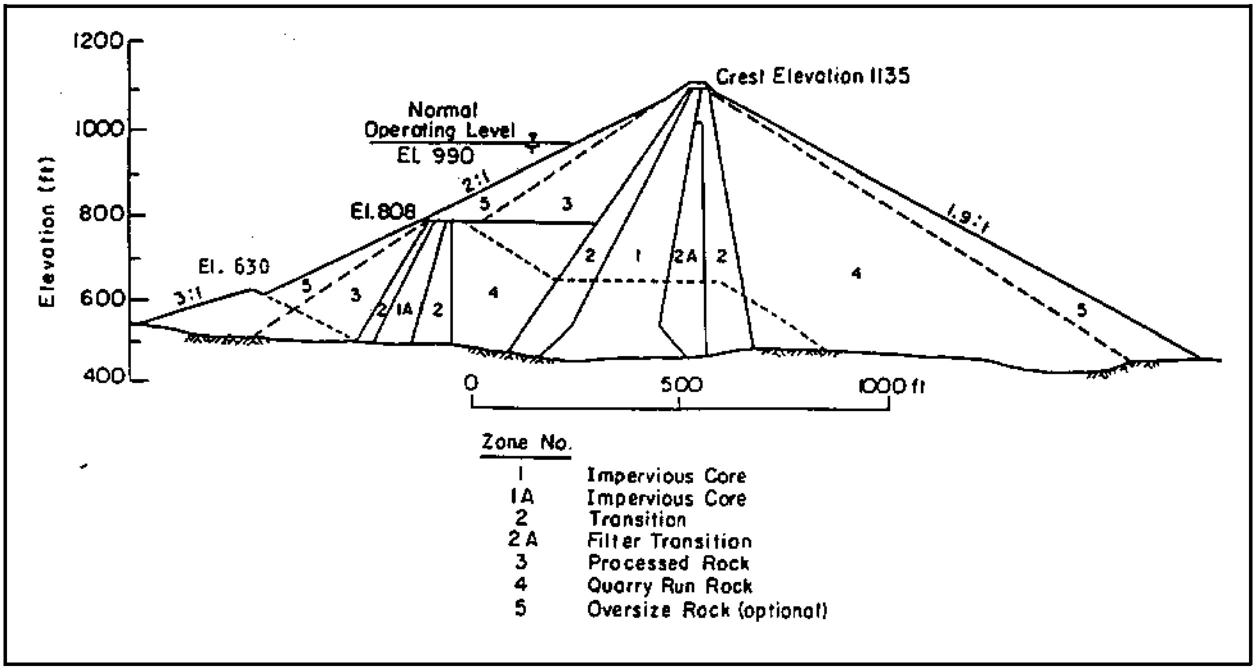

Figure 25. Cross-sectional view of New Melones Dam

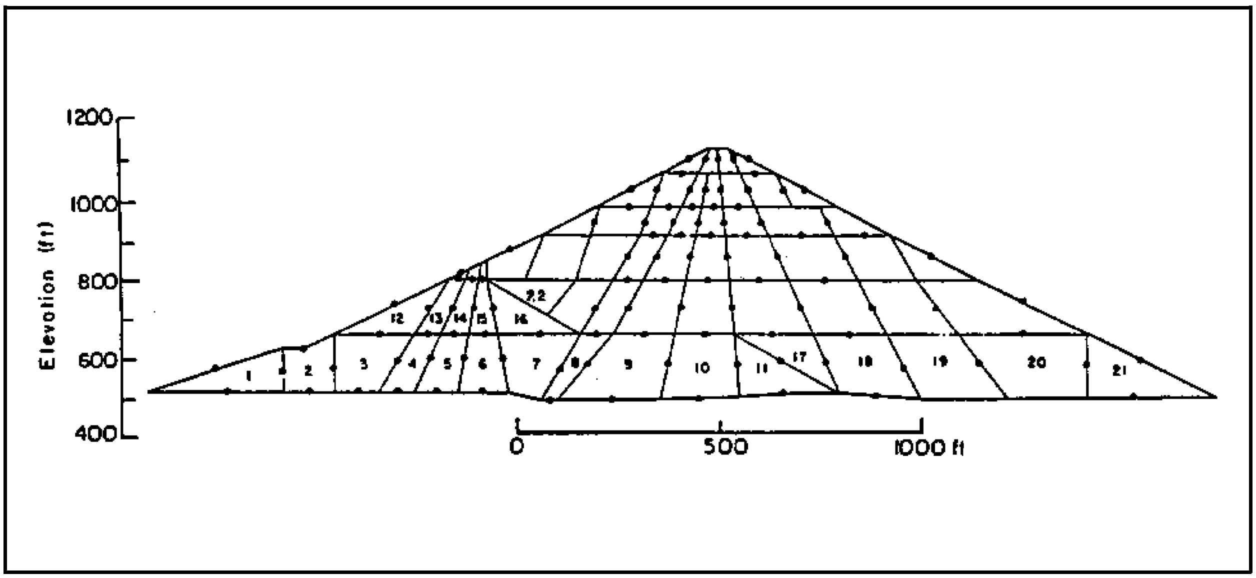

Figure 26. Finite element mesh of New Melones Dam

e. *Results*. The analysis was summarized providing answers to the three primary questions posed earlier as reasons for performing the study. Figures 28 and 29 show that the expected horizontal movements for the "stiff" and "soft" cores show that the maximum calculated horizontal movements during the development of steady state seepage was about 0.6 ft toward the downstream. The analysis indicated that some upward movement (rebound) might occur due to the effects of buoyancy. Upward movement (rebound) in the upstream shell may occur due to the effects of buoyancy as the upstream shell becomes submerged. However, it was reasoned that other effects such as creep or secondary compression which were not accounted for in the analysis would contribute to a net

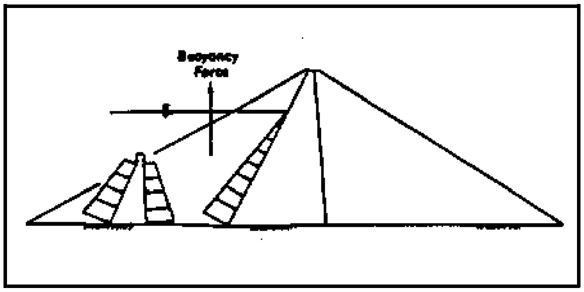

Figure 27. Treatment of upstream shell during reservoir filling

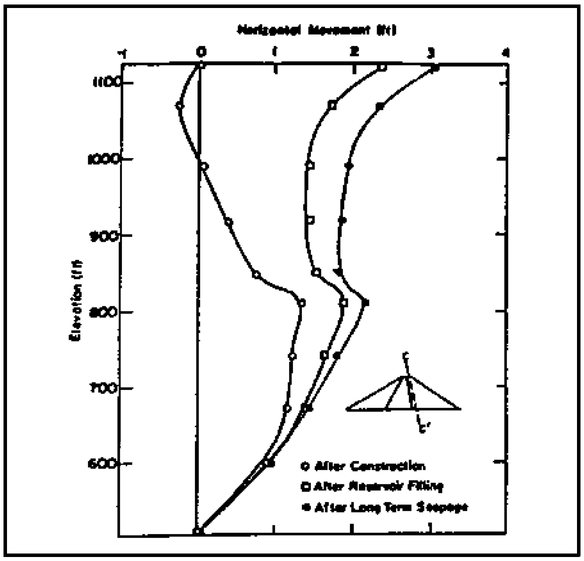

Figure 28. Horizontal movements for “stiff” core FE analysis

settlement rather than an uplift movement. The maximum and minimum principal stresses in the embankment for the “stiff” core case are shown in Figures 30 and 31, respectively. For both cases, the maximum and minimum principal effective stresses in the upstream shell decreased due to the effect of submergence. The maximum principal effective stress in the downstream shell decreased a small amount, and the minimum effective principal stress increased during consolidation and the development of long-term seepage in the core. As part of another finite element calculation, the long-term stresses were calculated using the hyperbolic constitutive model under the assumption that the construction was slow enough so as not to induce excess porewater pressures during the placement of fill.

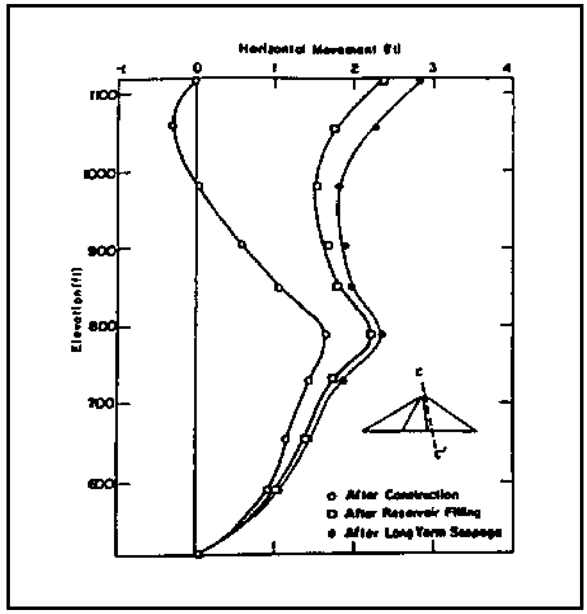

Figure 29. Horizontal movements for “soft” core FE analysis

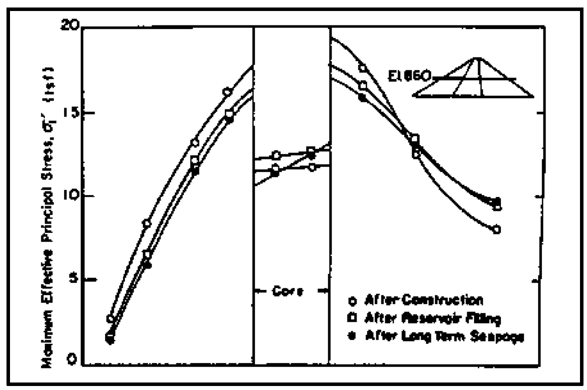

Figure 30. Maximum principal stress for “stiff” core analysis

These stresses were compared with the long-term stresses computed using the CON2D model in Figure 32. The results show that the stresses are nearly the same for this case. Overall, the movements in the embankment were considered small for the range of compaction conditions considered in the analysis. It was speculated that the movements would have been larger had the core been treated as a wetter and softer material. Additionally, the difference between the long-term stresses computed with the hyperbolic model and the consolidation model might also have been greater for the wetter and softer core.

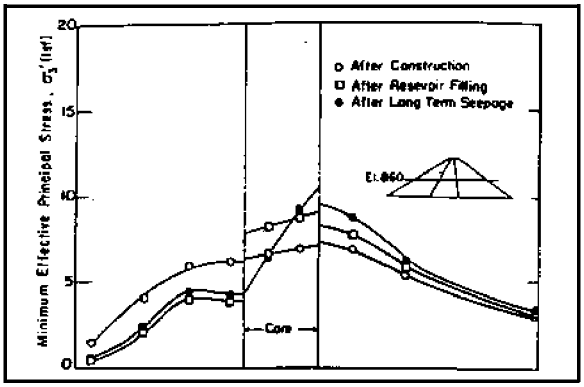

Figure 31. Minimum principal stress for "stiff" core analysis

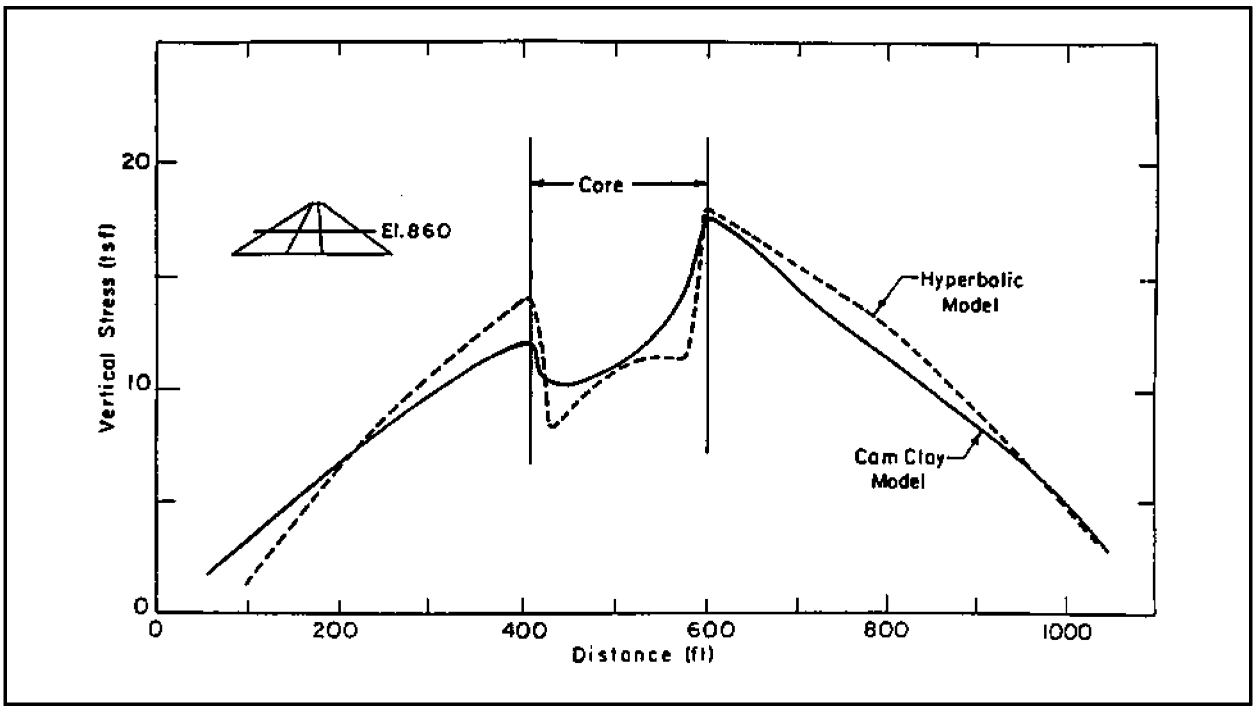

Figure 32. Comparison of long-term stresses for "drained" analysis with those for consolidation analysis